Pile tunnel interaction: Pile settlement vs Ground settlements

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ABSTRACT

The underground space of densely populated cities contains parts of buildings, utility installations, deep foundations, tunnels, and deep excavations. It is possible, and increasing more probable, that new underground constructions will be built within proximity of existing pile foundations. This paper analyses how a new a framework for pile analysis, a modified version of the load transfer method, can be used to predict the consequences of the tunnelling induced settlements on existing piles. The soil settlements are calculated with an analytical solution, and the pile settlements are calculated for different pile lengths, loading conditions, and distances between the pile and the tunnel centreline. The results indicate a wide range of possible pile settlements, in relation to the greenfield settlement trough. A simple interpretation scheme is used to understand the results, showing the importance of the profile of soil settlements along the pile.

Key Words: Deep Foundations, Tunnels, Pile Tunnel Interaction, Settlements

1. INTRODUCTION

The underground space of densely populated cities contains parts of buildings, utility installations, deep foundations, tunnels, and deep excavations. It is possible, and increasing more probable, that new underground constructions will be built near existing pile foundations. However, a great deal of uncertainty is still evident in regulations for minimum tunnel clearance and in the design of preventive measures against the effects of pile tunnel interaction. To face this problem, an extensive literature review was conducted by Dias and Bezuijen (2014a, 2015), and their main conclusions are detailed hereafter.

The case studies revealed that most structures were not damaged by a tunnel construction, but under certain conditions, preventive and active interventions were necessary. Overall, two mechanisms were described for the tunnel effects on piles: (A) The tunnel degrades the pile toe reaction, which requires a mobilization of shaft friction with limited settlements, and once the shaft is fully mobilized, higher settlements are necessary to recompress and mobilize the toe. (B) The toe reaction is not degraded, and the relative pile-soil settlements induce negative friction on the pile shaft, which increases the toe load.

Among the existing literature, there were no cases where the ultimate bearing capacity of a pile was actually tested before and after tunnelling. Several studies reported that the ratio between pile and ground-surface settlements depends on the pile position in relation to the tunnel; however, it is still debatable whether and how greenfield displacements are related to displacements in the presence of piles. The quantitative data revealed that, in terms of pile settlements, most piles did not reach failure, as defined by the limit of 10% the pile diameter. For piles located more than two tunnel diameters to the side of the tunnel alignment, the pile settlements were generally smaller than 1% of their diameter and 50% of the equivalent surface settlements.

Regarding the axial forces in the piles, it was established that the tunnel excavation induced compressive forces in non-loaded piles that increased to the tunnel depth and decreased at deeper levels. For loaded piles the results indicated a reduction of axial force when the piles were located directly above the tunnel, and an increase in piles to the side of the tunnel alignment. The increments of axial force and the pile settlements were inversely proportional to this lateral distance.

In terms of causes and effects, most studies relate this interaction to the fact that the construction of a tunnel results in ground movements and that these ground movements can influence how a pile transfers its load to the ground. Another important point is that these ground displacements can increase the mobilization of the shaft friction, and if the ultimate shaft capacity is reached, significant settlements occur to remobilize the pile toe resistance. These factors suggest that if a method is to be devised to estimate the consequences of pile tunnel interaction, it should be able to: (I) Consider the effects of ground settlements in the load distribution along the pile. (II) The possibility and consequences of full shaft mobilization (Dias 2017).

Past studies, focused on a simple version of requirement (I), have been able to reproduce the trends of pile/surface settlement ratios and increments of axial stress from experiments (Dias and Bezuijen 2014). Just recently, Dias and Bezuijen (2018) proposed a new framework, based on a modified version of the load transfer method, which can deal with both (I) and (II), for any loading state of a pile. This framework has been applied in detailed analyses of the pile tunnel interaction mechanism, for a wide range of conditions (Dias and Bezuijen 2017b,c,d). This study will focus on the relation between the surface settlements and the pile settlements induced by a tunnel excavation, which can be used as a first estimate for the settlements of a superstructure supported by piles at different positions in relation to the tunnel.

2. MODIFIED LOAD TRANSFER METHOD FOR PILE ANALYSIS

The load-transfer method, first proposed by Coyle and Reese (1966), calculates the load and settlement profiles along the pile through mobilization functions for the pile toe and at several points along the shaft. By imposing a toe displacement, vertical equilibrium of the pile segments can be iteratively calculated upwards until the pile head. The capacities of both the toe and the shaft must be described as functions of the local pile settlement and can be bound by the pile capacity (Poulos and Davis 1980). Heterogeneous ground profiles can be directly modelled by assigning different functions along the pile.

These mobilization functions have, for the most part, only been calibrated for pile loading. However, there are important mechanisms taking place through the unloading stage. Irreversible deformations and residual loads are important examples related to the plasticity of the pile-soil interface and the rebound of the pile toe. Moreover, by ignoring the unloading path the models predispose the range of possible solutions for equilibrium. Therefore, Dias and Bezuijen (2018) proposed two modifications to the general load transfer method: (a) Include a distinct unloading path in the load transfer functions, for both the shaft and the toe. (b) Change the variable of pile settlement for a relative pile-soil settlement, enabling the framework to consider the effects of ground displacements. The first point was adapted from the mathematical model of Massad (1995), which has been successfully applied to the analysis of bored and driven piles (Massad 2014; Viana da Fonseca et al. 2007). The second point has been proposed for the analysis of piles in interaction with deep excavations (Korff 2012).

The variable of relative pile-soil settlement ($\Delta\delta$) is defined as the difference between the pile settlement (δp) and the soil settlement (δs) at any point along the pile, and all settlements are assumed positive downwards. So, a negative $\Delta\delta$ means that the soil settles more than the pile in that point, developing a downward shear stress at the interface, also known as negative friction. At the pile toe, $\Delta\delta < 0$ indicates that there are no reaction forces from the toe, as the soil is not in contact with it. On the other hand, a positive $\Delta\delta$ is associated with upward shear, also called positive shaft friction, and an upward toe reaction. The displacements are always measured from the reference position of each point, calculated considering the pile head at the ground surface and uniform segments along the unstrained pile body.

A tri-linear mobilization model is assumed for the shaft friction (Figure 1). The interface shear stress can be mobilized both upwards and downwards, and it was assumed that in both directions the same absolute value is achieved at full mobilization (τ_{max}). Once full mobilization is reached the model is perfectly plastic, in the sense that the displacements can continue to develop without changes in the mobilized shear stress. The model defines a transition level of mobilization (τ_{ep}) from the elastic (*S1*) to the elastoplastic (*S2*) slopes, which are defined directly through the ratios of shear mobilization to relative displacement ($\tau/\Delta\delta$). If unloading occurs after the transition level, it develops through a distinct unloading slope (*S3*), until the transition level in the opposite direction.

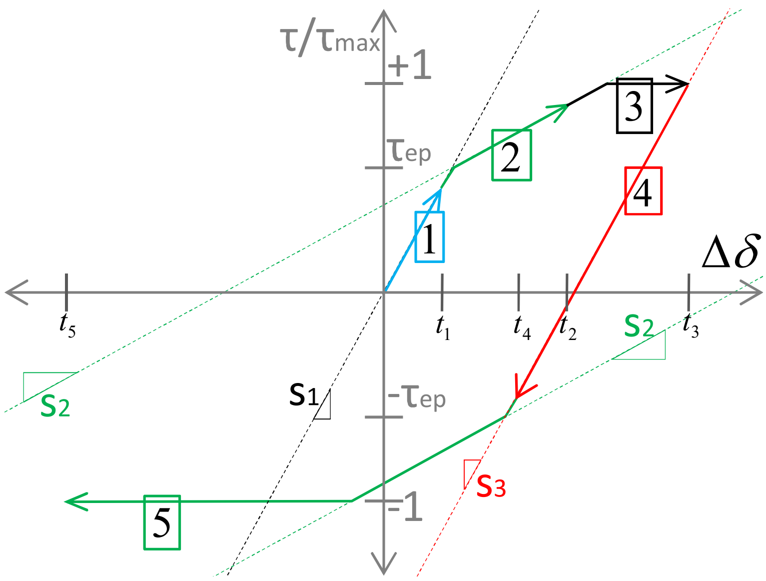

Figure 1. Tri-linear shaft mobilization model

For the pile toe reaction, a power function model is proposed, where mobilization only occurs for positive relative displacements (Figure 2). For the loading branch an exponential normalized function is defined, starting at the origin ($\Delta\delta$=0, q_b=0) and reaching full mobilization ($q_{b\ max}$) at a certain relative displacement, defined as $\Delta\delta^T$. The unloading branch has to be defined in a way that does not violate the restrictions of the domain, that is to say that it shouldn't calculate toe mobilization for $\Delta\delta < 0$. Considering this limitation and the large range of displacements for toe mobilization, a rebound factor (*Rb*) is used to define the range of unloading. Based on these functions, any new state of equilibrium can be determined through a root search process for the relative displacement at the pile toe that satisfies equilibrium and the load boundary conditions.

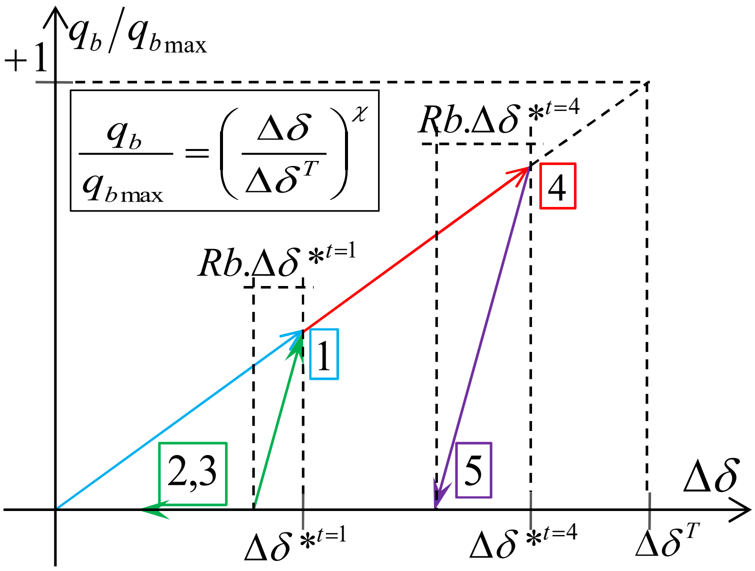

Figure 2. Toe mobilization model

3. A PILE'S NEW STATE OF EQUILIBRIUM IN INTERACTION WITH GROUND DISPLACEMENTS

In the framework of the modified load-transfer method, ground displacements act with the pile settlements to define the variable of relative displacements. Their balance sets the mobilization of the shaft and toe forces for equilibrium. At a certain depth, if the soil settlements are higher than the pile settlement, negative friction develops, increasing the axial force on the pile. If the soil settlements are smaller than the pile settlement, positive friction develops, reducing the axial force on the pile. The pile response due to ground displacements (GD) will always depend on the initial mobilization of the pile capacity and the associated settlements.

As an example, consider a 20 m long, 1 m in diameter, weightless pile supported only by friction. The material of the pile body has a Young's modulus of 10 GPa. The maximum shaft capacity of 1 MN is obtained with a constant shear resistance along depth, and a perfectly plastic mobilization model ($S1 = S3 = 0.1$; S2 = 0; $\tau_{ep} = 1$). For a load of 500 kN ($WL/UBC = 50\%$) the settlement at the pile head is 5 mm. A linear profile of ground displacements, with 10 mm of settlement at the pile head and 0 at the pile toe, is then imposed to the pile.

The profile of axial stresses (Figure 3) shows how the effects of the ground displacements can be calculated without violating the boundary conditions of the problem (fixed head load) or the vertical equilibrium ($\sigma^n = 0$). The increment of axial stress forms a sort of parabola with the vertex around half of the pile depth. This can be understood through the profiles of settlements and shear mobilization, presented in Figure 4. Before the ground displacements (P), the pile settlements were almost uniform with depth. In relation to null ground displacements (GD=0), this caused an almost uniform shear mobilization with depth. The imposition of the GD causes an additional 5 mm of settlement to the pile head. In relation to the linear profile of the GD, the pile settles the same as the ground at the surface, but the difference increases with depth. The new profile of shear mobilization is in direct relation to that difference, setting zero mobilization at the surface and practically full mobilization at the pile toe. When compared to the original profile of shear mobilization, this represents unloading in the top part of the pile and loading in the bottom part. This causes the axial stresses to increase until half of the pile depth, and decrease from there on, leading to a parabola of axial stress increments.

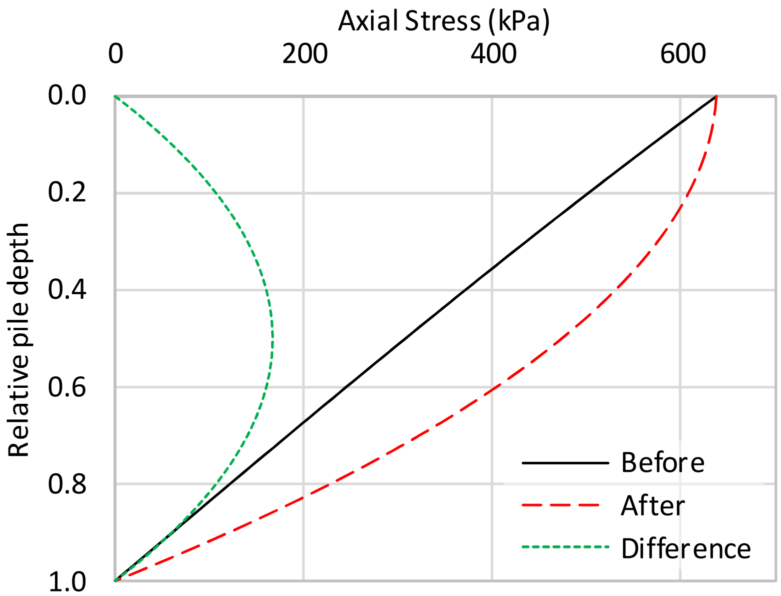

Figure 3. Example of friction pile in equilibrium with ground displacements: profiles of axial stress

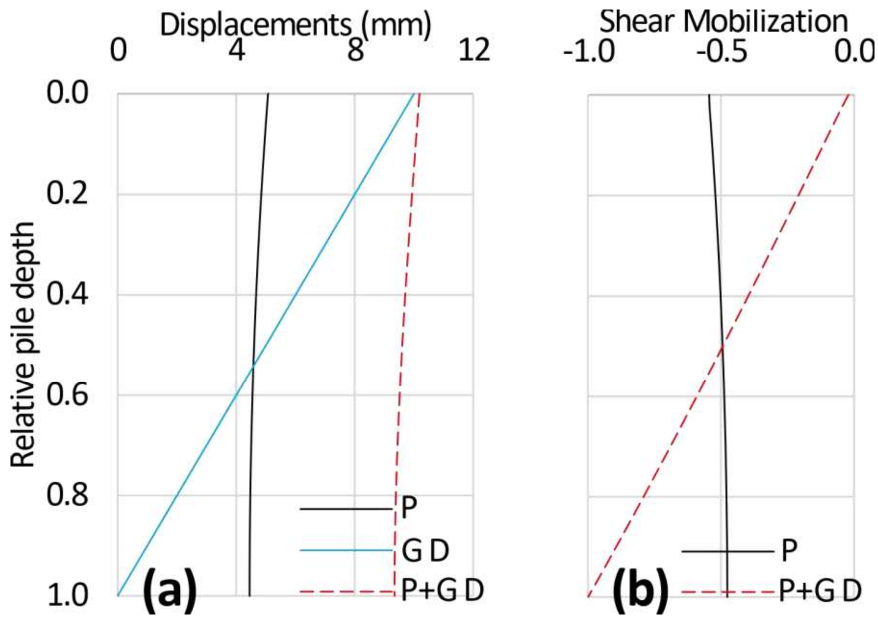

Figure 4. Example of friction pile in equilibrium with ground displacements: displacements (a) and shear mobilization (b).

This example demonstrates how a simple case of a pile in interaction with ground displacements requires the simultaneous consideration of several variables. It also shows that the mobilization models must be able to account for both loading and unloading to find the new state of equilibrium. The proposed framework can bring all these elements into the analysis and compute the consequences of any profile of ground displacements. Other basic examples of how the pile reacts to arbitrary profiles of ground displacements can be found Dias and Bezuijen (2017a).

4. PILE TUNNEL INTERACTION

The modified load transfer method can also be used with the settlements induced by a tunnel excavation. This study considers the analytical solution of Loganathan and Poulos (1998), which were derived for a homogeneous undrained clay layer, and assume that the lining is in contact with the tunnel invert, where there are no ground deformations. This is represented through an equivalent undrained ground loss that models the non-uniform radial convergence of the soil into the oval-shaped soil-lining gap. The ground settlement at any point (x, z) can be calculated with Equation 1.

$$\delta(x,z) = Vl.R^2.exp\left[\frac{-1.38x^2}{(Z_t+R)^2} - \frac{0.69z^2}{Z_t^2}\right].\left\{\frac{(Z_t-z)}{(z-Z_t)^2+x^2} + \frac{(Z_t-z)(3-4\nu)}{(z+Z_t)^2+x^2} + \frac{2z\left[(z+Z_t)^2-x^2\right]}{\left[(z+Z_t)^2+x^2\right]^2}\right\} \quad (1)$$

where *Vl* is the volume loss, *R* is the tunnel radius, *x* is the horizontal coordinate, *Zt* is the depth of the tunnel centre, *z* is the vertical coordinate, and *v* is the Poisson's ratio.

The following examples will consider 1 m in diameter, weightless, rigid piles, that are 10, 20, 30 or 40 m long. The unit shaft resistance is set to zero at the surface, and increasing with depth at a rate of 2 kPa/m. The unit toe resistance is set so that is represents 50% of the pile ultimate bearing capacity, which in the current layout follows the expression $q_{b\,max}$ *(kPa)* = $4.Zp^2$. The stiffness parameters for the load mobilization models described in Section 2 were defined as follows: Shaft: *S1* = *S3* = 0.050; *S2* = 0.025; τ_{ep} = 0.50 Toe $\Delta\sigma_T$ = 100 mm, χ = 0.3 and *Rb* = 0.2. With the same stiffness parameters, and ratios between Toe and Shaft capacities, all piles will follow the normalized load-settlement curve for a full load-unload cycle presented in Figure 5.

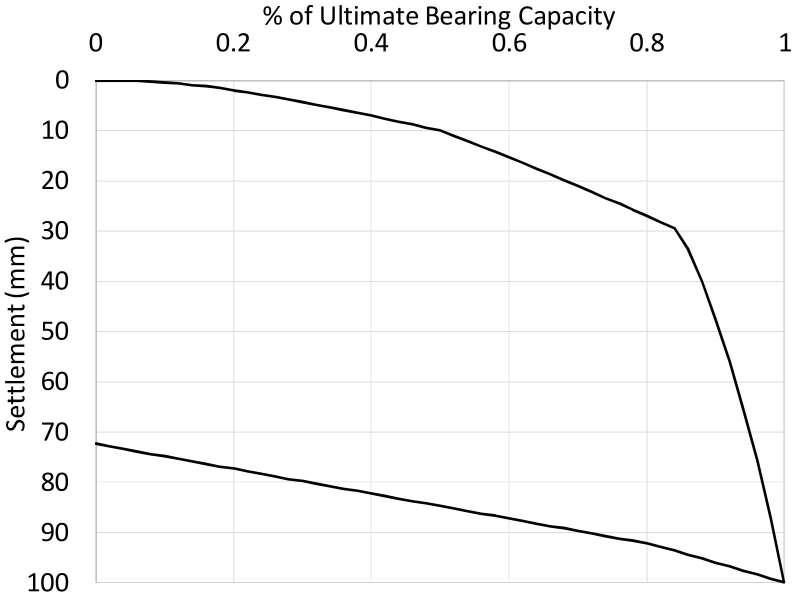

Figure 5. Normalized load settlement curve for all piles.

The ground settlements are calculated with Equation 1 for a tunnel with a 5 m radius, centred at a depth of 30 m, considering a volume loss of 1%, and a Poisson ratio of 0.3. The pile tunnel interaction will be calculated for the four pile lengths (10, 20, 30, 40 m), considering two initial loads in the piles: 25% and 75% of their ultimate bearing capacities, and along a wide range of horizontal distances between the pile alignment and the tunnel centreline: from 0 to 35 m. The calculated ground settlements are shown in Figure 6 for different lateral distances (*Ld*), normalized by the tunnel diameter (*Dt*). For lateral distances smaller than the tunnel radius (*Ld*<*0.5Dt*) the profiles pass through the tunnel, where the calculated values have no physical significance.

Two things should be noted while analysing these profiles of soil settlement along depth. The first is that until 10-20 m depth, the profile of settlements is almost constant, especially for small lateral distances. The second is that below the tunnel invert, located at 35 m, the soil movements are upwards, until a lateral distance of about 1 tunnel diameter

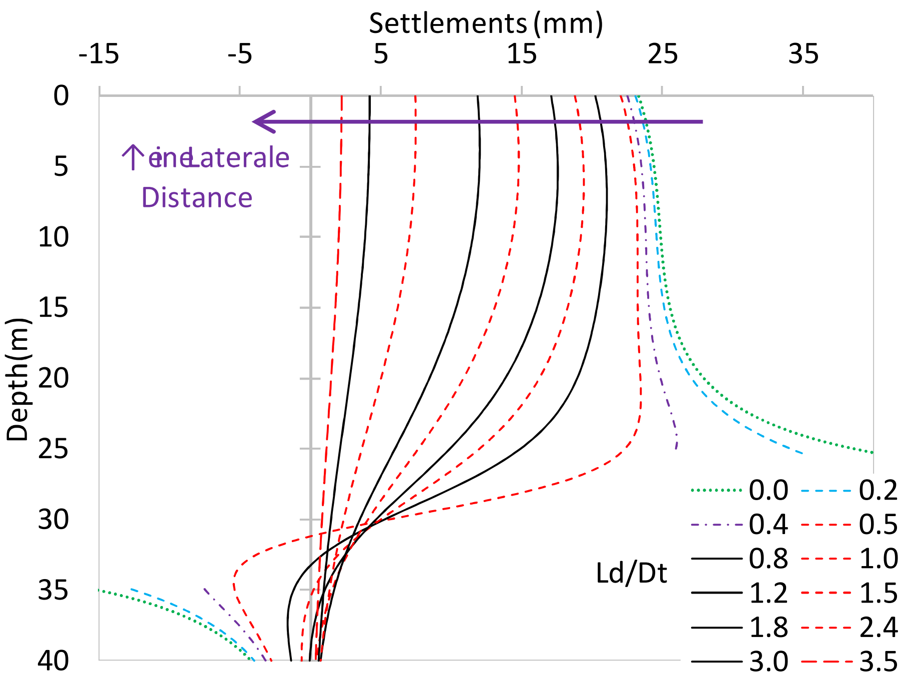

Figure 6. Ground settlements at different lateral distances until a depth of 40 m

The incremental pile settlement (after tunnelling minus before tunnelling) for the four pile lengths (10, 20, 30 and 40 m), at two loading states (25 and 75% UBC), for a range of lateral distances from 0 to 3.5 Dt, can be seen in Figure 7 together with the greenfield soil settlement trough. These results can also be analysed by the ratio between pile and soil surface settlement, as can be seen in Figure 8.

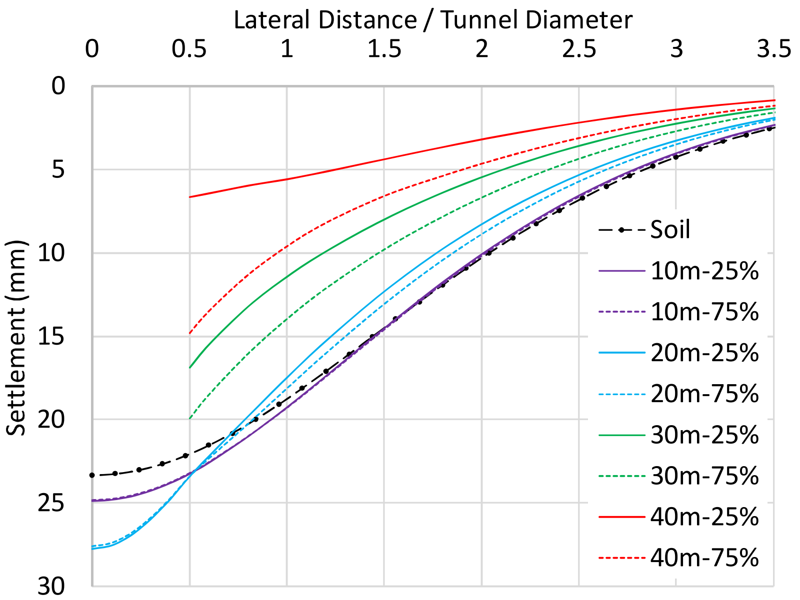

Figure 7. Pile settlement and soil settlements for different pile lengths, loading conditions, and relative positions

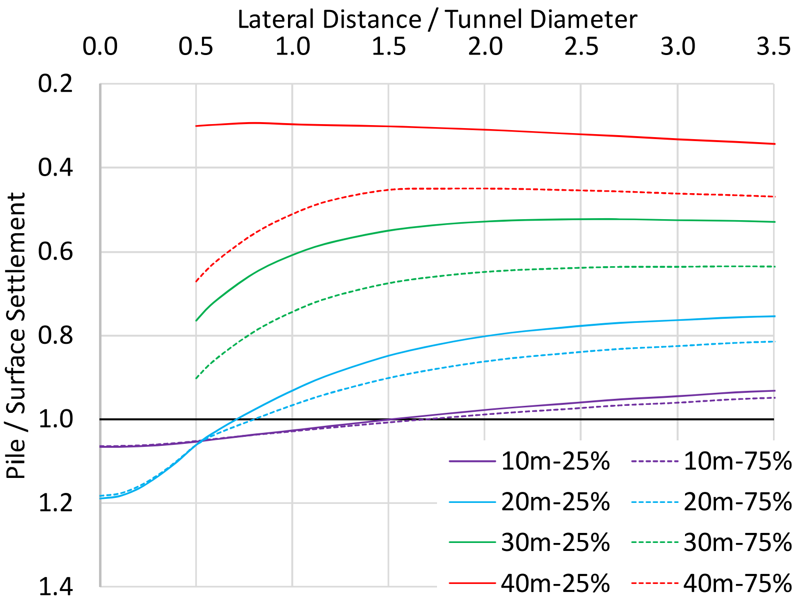

Figure 8. Ratio between pile and soil settlement for different pile lengths, loading conditions, and relative positions

The two shorter piles, which could be above the tunnel, suffered higher settlements than the longer ones. These short piles settled even more than the soil between [0; 0.7Dt] for the 10 m pile, and [0; 1.5Dt] for the 20 m pile. Another distinction of the shorter piles is that the effect of the working load was very small when the pile was right above the tunnel, increasing a little for larger distances. The longer piles settled less than the soil surface for all lateral distances and loads, but for them, the effect of the working load was much more pronounced. The more loaded piles were more susceptible to the tunnelling induced settlements. In the studied range, the longer the pile, the less it settled due to tunnelling, except for piles placed right above the tunnel, where the longer pile settled more.

These pile settlements are the result of a complex interaction of load mobilization as a function of relative displacements, which depend on the tunnelling soil settlements, for both the pile toe and along the pile shaft. However, to make sense out of these results, one can think of a simple scheme: the pile settles to accommodate the soil movements at the pile toe. If the settlements along the shaft and at the toe level are similar, there is no redistribution of forces, as the new equilibrium is basically a translation of the previous one. However, if the settlements along the shaft are higher, they will pull the pile down, redistributing the total load between toe and shaft forces.

Following this scheme, the shorter piles (10 and 20 m) settled more than the longer ones (30 and 40 m) simply because the soil settlement at the depth of their toes was higher, as can be seen in Figure 6. The same principle applies for the inverse relation that occurs right above the tunnel between the two shorted piles, because in that region the soil settlements increase with depth, so the toe of the 20 m pile right above the tunnel was under higher settlements than the toe of the 10 m pile.

The shape of the soil settlements with depth can also explain why the pile loading had such a small effect for the shorter piles: They were placed in a region where the soil settlements are almost constant with depth, so that their settlements represent a simple translation to a new position, without any stress redistribution. This Is less accurate for higher lateral distances, where the settlements start to vary a little more, which is also reflected on the pile settlements.

For the longer piles that was not the case. The settlement at the level of their toes was much smaller than the surface settlements, which led to a redistribution of forces along the pile. The higher settlements for the more loaded piles can be understood through their load-mobilization functions. The stiffness of both the toe and the shaft decreased with loading, making the more loaded piles more susceptible to settlements. If, for example, all the elements had been modelled as linear springs, the induced settlements wouldn't be different for piles under different loads.

5. CONCLUSION

This paper presented how a modified version of the load transfer method can be used to predict how a pile reacts when subjected to tunnelling induced soil settlements. A series of examples of pile tunnel interaction have been used to analyse the relation between the incremental pile settlements and the greenfield settlement trough. The relation between the two varies widely, with piles settling between 30 and 120% of the greenfield soil settlements at the surface. A simple interpretation scheme has been discussed to analyse the pile reactions as a function, primarily, of the soil settlements along depth. As valuable as this interpretation scheme is, it is not enough to obtain quantitative results of the complex process of pile tunnel interaction.

All the results discussed in this paper are only valid for the profiles of soil settlements with depth that have been calculated with Equation 1. Any soil conditions or tunnelling processes that cause a different field of soil deformation, will also cause different results of pile tunnel interaction. This raises two important points: 1. The importance of being able to predict not only the settlement trough, but also the settlements along depth, caused by any tunnel excavation. 2. The flexibility of the proposed framework to deal with any profile of soil settlements, calculated analytically, numerically, or even measured in the field.

ACKNOWLEDGEMENTS

The first author acknowledges the financial support of the Brazilian Research Agency CNPq.

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